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Making unsaturated soil mechanics accessible for engineers: Preliminary hydraulic–mechanical characterisation & stability assessment

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ABSTRACT

The paper presents an accessible approach for preliminary hydraulic and mechanical characterisation of unsaturated soils and stability analysis of geo-structures under unsaturated conditions. The approach is 'accessible' in the sense that the laboratory investigation and geotechnical analyses can be completed in a relatively short time and at a reasonable cost. The overall philosophy behind this approach is that laboratory testing should not be eliminated but kept to a minimum and that any simplification introduced should be conservative. The paper illustrates that shear strength can be characterised conservatively by performing constant water content tests with no facilities to control/monitor suction and that water retention behaviour can be predicted successfully by performing a single water retention measurement. The paper also demonstrates that stability analyses can be carried out within the classical framework of limit analysis. We show that a preliminary analysis of the influence of infiltrating rainwater on suction can be carried out using a simple and conservative method that makes use of solutions available in 'saturated' geotechnical textbooks. Such an approach may be used to assess when it is worthwhile or necessary to conduct further time consuming and costly unsaturated experimental testing, field monitoring and numerical analyses when considering the stability of geostructures.

1. Introduction

Geotechnical structures like embankments, cuttings, earth dams, retaining structures, slopes, and foundations are very often located above the groundwater table and their response therefore involves layers with negative pore-water pressures, which are generally unsaturated. The effect of negative pore-water pressure is very often neglected in engineering practice because either i) the ground water table is assumed conservatively to be at the ground surface or at the surface of the retained material (e.g. shallow and deep foundations, retaining structures) or ii) the soil above the water table/phreatic surface is assumed to be dry in the sense that pore-pressure is set to zero in the analyses (e.g. back-analysis of failures, embankments).

Assuming that the ground water table is at the ground surface can result in significant over-design of engineering structures, and cost savings could be possible if a proper assessment of the unsaturated behaviour and hydraulic boundary conditions (e.g. rainfall) was made. In many cases, cost savings would also be accompanied by a reduction in the energy and carbon 'embodied' in the geotechnical structure.

On the other hand, the design of remediation and upgrade measures for geotechnical structures is often based on the back-analysis of existing geotechnical structures, which can be misleading if the influence of suction above the water table is ignored. For example, steep artificial and natural slopes, including vertical and near-vertical cuts, are often stable because of the suction generated by partial saturation. If this effect is not understood and quantified, an effective cohesion can misleadingly be assigned to the soil.

Overall, neglecting the unsaturated aspects of behaviour can lead to problems associated with ground–atmosphere interactions being overlooked, which may be costly to remedy at a later stage. These include foundation subsidence associated with shrinkage after prolonged periods of dry weather, heave in swelling soils, volumetric collapse of meta-stable open soils, and instability of natural and artificial slopes triggered by rainfall.

To implement unsaturated soil concepts into current geotechnical practice, simple tools need to be developed for 'preliminary' assessment of the effect of suction on the response of geostructures. As a first step in this direction, this paper focuses on the pre-assessment of stability of 'routine' geotechnical structures in unsaturated soils based on simple analytical procedures using a very limited set of laboratory data. This approach is 'accessible' in the sense that the laboratory investigation and geotechnical analyses can be completed in a relatively short time and at a reasonable cost. The overall philosophy is that laboratory

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testing should not be eliminated but kept to a minimum and that any simplification introduced in data interpretation and analyses should be conservative. This approach is intended to be used to identify (and not replace) when a more accurate design based on appropriate laboratory testing, field monitoring, and numerical analysis is required.

2. An accessible and inexpensive approach to unsaturated shear strength characterisation

A shear strength equation is required to analyse the stability of geotechnical structures. There is no unanimous consensus concerning the relationship to model the effect of partial saturation on the shear strength of unsaturated soils. Two shear strength criteria are therefore identified and validated in this paper, which will serve as the basis to discuss an approach for a low-cost estimation of unsaturated shear strength.

For the sake of simplicity, the paper will focus on the critical (ultimate) shear strength, which in general enables a conservative estimate of the factor of safety against instability to be made. It is assumed here that the parameters characterising the shear strength at the critical state are independent of void ratio, similar to saturated soils.

2.1. Ultimate (critical) shear strength criteria for unsaturated soils

Shear strength criteria for unsaturated geomaterials proposed in the literature generally fall into two broad categories. The first approach assumes that the contribution of partial saturation to shear strength is only dependent on suction (Fredlund et al., 1978; Alonso et al., 1990; Khalili and Khabbaz, 1998). In this case, shear strength is expressed by Eq. (1)

$$\tau = \sigma \tan\phi' + \Delta\tau(s) \quad \text{or} \quad q = \mathrm{M}p + \Delta q(s) \tag{1}$$

where τ is the shear strength, σ is the total stress normal to the failure plane, ϕ' is the 'saturated' angle of shearing resistance, $\Delta\tau$ is the contribution of partial saturation to shear strength, s is the suction, q and p are the deviator and total mean stress at the ultimate state respectively, M is the slope of the 'saturated' critical state line, and Δq is the contribution of partial saturation to deviator stress at the ultimate state.

The second approach assumes that the contribution of partial saturation to shear strength is independently controlled by both suction and degree of saturation (Toll, 1990; Vanapalli et al., 1996; Öberg and Sällfors, 1997) according to Eq. (2)

$$\tau = \sigma \tan\phi' + \Delta\tau(s, S_r) \quad \text{or} \quad q = \mathrm{M}p + \Delta q(s, S_r) \tag{2}$$

where S_r is the degree of saturation.

Eq. (2) suggest that different degrees of saturation at given level of suction lead to different values of ultimate shear strength. Different degrees of saturation may occur at the same suction because of the effect of void ratio on the water retention curve (Romero and Vaunat, 2000; Karube and Kawai, 2001; Gallipoli et al., 2003) or hydraulic hysteresis. The variation in ultimate shear strength at a given suction associated with a change in the degree of saturation may be significant. Fig. 1 presents the contribution of partial saturation to ultimate shear strength for reconstituted BCN clayey silt samples (Boso, 2005). Three series of samples were tested in the direct shear box. Each series was obtained by consolidating samples from a slurry state to different vertical stresses, σ_{cons} (100, 300, and 500 kPa). Samples from each series were subjected to different values of suction by air-drying to target water contents; samples were subsequently sheared in a suction-monitored shear box. The three different initial pre-consolidation stresses generated different initial void ratios and, hence, three different water retention curves (Tarantino, 2009). As a result, samples from these three different series showed, at the same suction, different degrees of saturation at the ultimate state as shown in Fig. 1a. This resulted in significantly different critical state shear strength at a given suction. It is worth noticing that if shear strength data are plotted against the product of suction s times the degree of saturation S_r as shown in Fig. 1b, differences tend to disappear suggesting that suction and degree of saturation independently control shear strength according to Eq. (2).

The independent effect of suction and degree of saturation is also illustrated in Fig. 2a, where the contribution of partial saturation to the ultimate deviator stress Δq is plotted against suction for compacted kaolin samples dried from a saturated state or wetted after being dried to a given degree of saturation (Uchaipichat, 2010). At the same suction, the degree of saturation is higher along a drying path than on a wetting path (hydraulic hysteresis) and this results in a higher deviator stress contribution as shown in Fig. 2a. Again, if the contribution to deviator stress Δq is plotted against suction weighted by an 'effective' degree of saturation, differences between samples on the drying and wetting curve disappear (Fig. 2b).

The concept of effective degree of saturation has been discussed by Tarantino and Tombolato (2005) and Tarantino (2007). They observed that the shear strength of unsaturated compacted clay could be modelled using the following equation:

$$\tau = (\sigma + sS_{rM}) \tan\phi' \quad \text{or} \quad q = M(p + sS_{rM}) \tag{3}$$

where the contribution of partial saturation to shear strength is proportional to suction weighted by the degree of saturation of the macropores, S_{rM}. This is defined as:

$$S_{rM} = \frac{e_w - e_{wm}}{e - e_{wm}} \tag{4}$$

where e is the void ratio, e_w is the water ratio (i.e. e_w is the volume of water per volume of solids, $e_w = wG_s$), and e_{wm} is the microstructural water ratio, which is associated with the adsorbed water stored in the saturated aggregates (Eq. (4) is only valid for $e_w > e_{wm}$). The rationale behind this approach is that only capillary water in the macropores, i.e. the water in the pore space between the aggregates, contributes to shear strength. Tarantino and Tombolato (2005) determined the microstructural water ratio e_{wm} as a best-fit parameter using Eqs. (3) and (4) and found this value to be in very close agreement with the microstructural water ratio inferred from the measured pore size distribution (Tarantino and De Col 2008). A similar idea can be found in the Soil Science literature. Nearing (1995) analysed unconfined compression data on compacted 'aggregated' clay and also speculated that shear strength is only controlled by inter-aggregate water.

An approach based on an 'effective' degree of saturation was also presented by Vanapalli et al. (1996):

$$\tau = \left(\sigma \qquad\right) \tan\phi' \quad \text{or} \quad q = \mathrm{M}\left(\qquad \right) \tag{5}$$

where the degree of saturation is scaled through the exponent k. Eqs. (3) and (5) suggest that only one parameter is sufficient to model unsaturated shear strength, in addition to the 'saturated' parameters and can therefore in principle be determined by performing a single shear test. To assess the capability of Eqs. (3) and (5) to model the shear strength of unsaturated soils, 18 datasets were examined in this paper. Reconstituted, compacted and natural soils were considered with grain size distributions ranging from clayey to sandy soils (Table 1). Parameters e_{wm} and k were obtained by best-fitting using the least square method and are also reported in Table 1. Ultimate (critical) state data were considered.

The validation of Eqs. (3) and (5) is presented in Figs. 3 and 4. It can be observed that the two models fit the experimental data in a very similar way and that both models perform quite well. These two models are indeed very similar at relatively high degrees of saturation as discussed by Alonso et al. (2010). It is interesting to note that when the clay fraction is negligible or absent and little or no aggregation is expected, data are fitted by $e_{wm} = 0$ and $k = 1$ (Fig. 4). This corroborates the

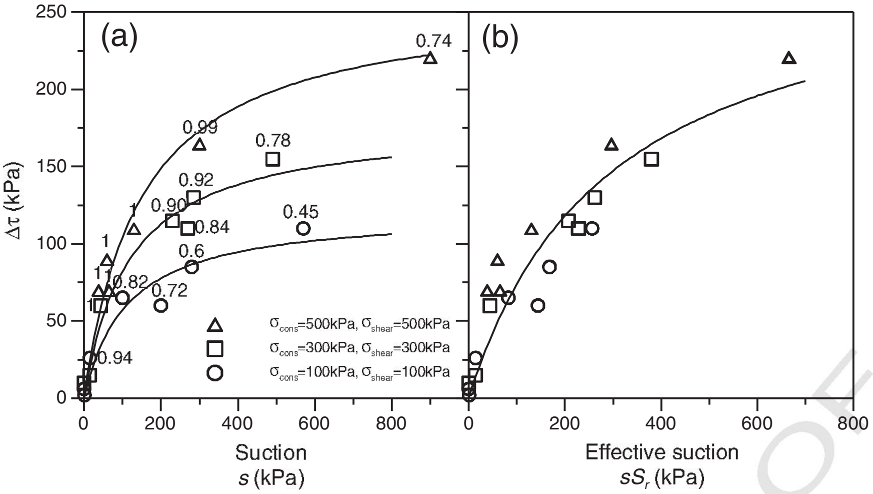

Fig. 1. Effect of degree of saturation on ultimate shear strength of reconstituted BCN silt (samples consolidated at the vertical stress σ_{cons}, air-dried and sheared at the vertical stress σ_{shear}). The degree of saturation is labelled by each data point in (a).
After Boso (2005).

assumption that the parameters e_{wm} and k account for the intra-aggregate water. This point is reinforced by Fig. 5, which shows that the microstructural water ratio e_{wm}, as inferred from shear strength data, correlates quite well with the clay fraction (the trend lines are only drawn to help visualise the data and are not intended to suggest any empirical correlation). There is an exception in Fig. 5 given by the data point from Toll (1990) which is due to the "gravel" portion being composed of iron-cemented fine material.

2.2. Accessible characterisation of unsaturated shear strength using a 'conventional' constant water content test

It has been shown that shear strength can be modelled by using either Eq. (3) or (5). Therefore in order to characterise the unsaturated shear strength, a single parameter, e_{wm} (Eq. (3)) or k (Eq. (5)) needs to be determined experimentally, in addition to the saturated effective shearing resistance ϕ' or critical state parameter M.

Suction-controlled or suction-monitored direct shear or triaxial tests are time consuming, make use of non-conventional equipment, and require specific experimental expertise. They are therefore not currently suitable for routine engineering applications. On the other hand, one of the simplest tests that can be carried out in a conventional laboratory is a constant water content shear test using either the shear box or the triaxial cell with no specific facilities for unsaturated testing. The use of this test for a conservative estimate of the parameters e_{wm} or k is discussed in this section.

To this end, hydraulic paths in terms of suction and 'effective' degree of saturation during the water-undrained compression and shearing stage from tests published in the literature are examined. Hydraulic paths followed during the water-undrained compression stage are shown in Fig. 6a for a reconstituted clayey sandy silt (Boso, 2005). Samples were first normally consolidated from slurry to 500 kPa vertical stresses, air-dried to target water content, and then compressed under water-undrained conditions. It can be observed that during constant water content compression the product of suction times the effective degree of saturation, sS_r^k tends either to decrease or remain nearly constant (contours of sS_r^k = constant are shown in the figures). The same conclusion is drawn by considering the water-undrained compression paths for compacted clay (Fig. 6b) and compacted clayey silt (Fig. 6c).

Hydraulic paths recorded upon water-undrained shearing are shown in Fig. 7a for a reconstituted clayey sandy silt (Boso, 2005), compacted clayey silt (Jotisankasa, 2005), compacted sandy clay

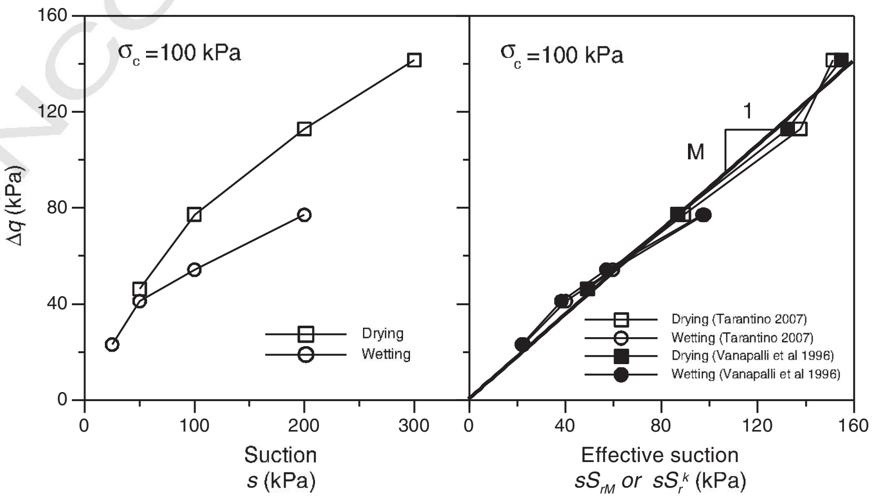

Fig. 2. Effect of hydraulic hysteresis on ultimate shear strength of compacted kaolin. (a) Shear strength against suction; (b) shear strength against suction weighed by an 'effective' degree of saturation.
After Uchaipichat (2010).

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Table 1
Shear strength datasets analysed and modelling parameters.

Soil	Clay	Silt	Sand	w_P	w_L	CS/CW	e_{wm}	k
Compacted clay (Uchaipichat, 2010)	0.60	0.40	–	0.31	0.52	CS	0.87	9.40
Natural silty clay (Kayadelen et al., 2007)	0.67	0.28	0.05	0.32	0.77	CS	0.63	3.74
Compacted gravel Toll (1990)	0.09	0.05	0.26	0.29	0.61	CW	0.52	6.82
Compacted clay (Tarantino and Tombolato, 2005)	0.8	0.2	–	0.32	0.64	CW	0.4	2.23
Compacted sandy clay (Rahardjo et al., 2004)	0.42	0.24	0.34	0.22	0.36	CW/CS	0.37	6.80
Compacted sandy clay (Toll and Ong, 2003)	0.41	0.21	0.38	0.22	0.36	CW	0.31	3.04
Compacted silty sand (Rampino et al., 2000)	0.18	0.20	0.62	0.21	0.35	CS	0.18	2.42
Compacted clayey silt (Maâtouk et al., 1995)	0.18	0.66	0.16	0.15	0.22	CS	0.17	2.83
Compacted agricultural soil (Adams and Wulfsohn, 1998)	0.28	0.24	0.48	0.18	N/A	CW	0.15	1.30
Reconstituted sandy silt (Wang et al., 2002)	0.58	0.42	0.14	0.29	CS	0.15	1.87	
Compacted clayey silt (Jotisankasa et al., 2009)	0.26	0.52	0.22	0.18	0.28	CW	0.15	1.68
Compacted clayey sandy silt (Barrera, 2002; Buenfil, 2007)	0.18	0.42	0.40	0.16	0.32	CW/CS	0.12	1.49
Reconstituted clayey sandy silt (Boso, 2005)	0.18	0.42	0.40	0.16	0.32	CW/CS	0	1
Compacted silt (Thu et al., 2006)	0.15	0.85	–	0.36	0.51	CW	0	1
Compacted silt (Capotosto and Russo, 2011)	0.10	0.90	–	0.15	0.22	CS	0	1
Natural silty sand (Papa et al., 2008)	0.03	0.35	0.62	–	–	CS	0	1
Natural & reconstituted silty sand (Cattoni et al., 2007)	0.03	0.27	0.70	–	–	CS	0	1
Reconstituted silt (Geiser et al., 2006)	0.08	0.72	0.20	0.17	0.25	CS	0	1

CW = Constant water content shearing; CS = Constant suction shearing.

(Toll and Ong, 2003) and compacted sandy clay (Rahardjo et al., 2004). In general, the change of sS_r^k upon shearing is not intuitive. Dilatant behaviour is associated with a decrease in 'effective' degree of saturation (water content is constant) and therefore one would expect suction to increase. This is not always the case as it can be observed that dilatant behaviour is often associated with a decrease in suction (Fig. 7b, c, d). Similarly, contractile behaviour (increase in degree of saturation) can be associated with either a decrease or an increase in suction (Fig. 7b).

Notwithstanding the various responses observed upon shearing, it can again be observed that product sS_r^k tends either to decrease, or remain nearly constant but it never increases significantly upon water-undrained shearing.

The evolution of the product sS_r^k upon water-undrained compression and shearing has a very interesting implication in shear strength modelling. Fig. 8 shows qualitatively the shear strength failure envelope in terms of Δq against the product of suction and degree of saturation, sS_r^k. Suction and degree of saturation characterising the shear strength are those at ultimate (critical) state. However, these variables would never be measured using a 'conventional' direct shear or triaxial cell. The only information that would be available is the initial suction and degree of saturation, before the sample is compressed and sheared at constant-water content. The initial value of the product sS_r would be higher than its value at the ultimate state as shown in Figs. 6 and 7. If the shear strength measured in a conventional apparatus is attributed to the initial value of sS_r (rather than to its ultimate value), then a conservative estimate of shear strength can be made as shown in Fig. 8.

The practical procedure for the conservative estimate of shear strength is illustrated below. It is developed with reference to Eq. (5) but can equally be applied to Eq. (3).

1) The prediction of unsaturated shear strength requires the determination of the parameter k; which should be determined by measuring relevant variables at the ultimate state;
2) k_0 is the conservative estimate of k, which will be obtained from the measurement of initial suction, $s_{0,}$ and degree of saturation, $S_{r0,}$ of the sample before constant water content compression and shearing;
3) Consider a sample having initial degree of saturation S_{r0} and initial suction s_0 inferred from the water retention curve (determined according to the procedure illustrated in the next section);
4) Let the sample be compressed and sheared under constant water content and be σ and τ the normal and the shear stresses respectively measured at the ultimate state (or p and q the isotropic and deviator stresses respectively measured at the ultimate state);
5) By rearranging Eq. (5), the parameter k_0 can be obtained as follows:

$$k_0 = \frac{\log\left(\frac{\tau-\sigma \quad \phi'}{\phi'}\right)}{\log S_{r0}} \quad \text{or} \quad k_0 = \frac{\log\left(\frac{-}{\quad}\right)}{\log S_{r0}}. \tag{6}$$

In principle, one single test would be sufficient to estimate k_0. However it would be good practice to duplicate or triplicate the measurement at different values of σ or p;
6) If the degree of saturation S_r and suction s at the ultimate state were measured, the parameter k could have been determined using equations similar to Eq. (6) with k, s, and S_r in place of k_0, s_0, and S_{r0}. According to Fig. 9, it must be $k_0 \geq k$.

An example of the estimation of the shear strength envelope based on the initial value of s_0S_{r0} (rather than its ultimate value), is shown in Fig. 9a for data from Jotisankasa et al. (2009). According to Eq. (6), the parameter k_0 is the slope of the line interpolating the data in the plane where the horizontal and vertical axes are represented by the denominator and numerator respectively in Eq. (6). For comparison, the calculation of k is also reported in Fig. 9a. In this case, k and k_0 are found to be equal. Another two examples are given in Fig. 9b for the reconstituted clayey sandy silt (Boso, 2005) where $k_0 > k$ and in Fig. 9c for compacted clay (Tarantino and Tombolato, 2005) where $k_0 = k$.

3. An accessible and inexpensive approach to water retention characterisation

Water retention behaviour, i.e. the relationship between degree of saturation and suction, controls shear strength (Eq. (3) or (5)) and water flow and needs to be identified in a stability analysis containing unsaturated soils. Due to the time consuming and costly nature of measuring water retention curves in the laboratory, the estimation of water retention behaviour from basic soil properties has long been a topic of interest, particularly within the soil science community (Cornelis et al., 2001). Three different methods for the estimation of the water retention behaviour are examined in this paper. The first two methods are based on the prediction of water retention curve parameters from soil properties by regression analysis (Vereecken et al., 1989; Chin et al., 2010) whereas the third approach makes use of a physical–

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conceptual model to estimate the water retention curve (Arya and Paris, 1981).

3.1. WRC estimation approaches

The method presented by Vereecken et al. (1989) estimates the parameters of the van Genuchten (1980) equation for the soil water retention curve using information on the grain size distribution, dry density (or void ratio) and carbon content. Chin et al. (2010) predicts the parameters of the Fredlund and Xing (1994) equation using void ratio and the measured water content (or degree of saturation) at a particular value of matric suction, and is referred to as a 'one-point method'. Although a number of one-point estimation methods have been proposed in the literature (Vanapalli and Catana, 2005; Catana et al., 2006; Houston et al., 2006), the method outlined by Chin et al. (2010) was selected as it uses fewer parameters than other ones. The Arya and Paris (1981) method requires a detailed description of the grain-size distribution, void ratio and a parameter α. Arya et al.

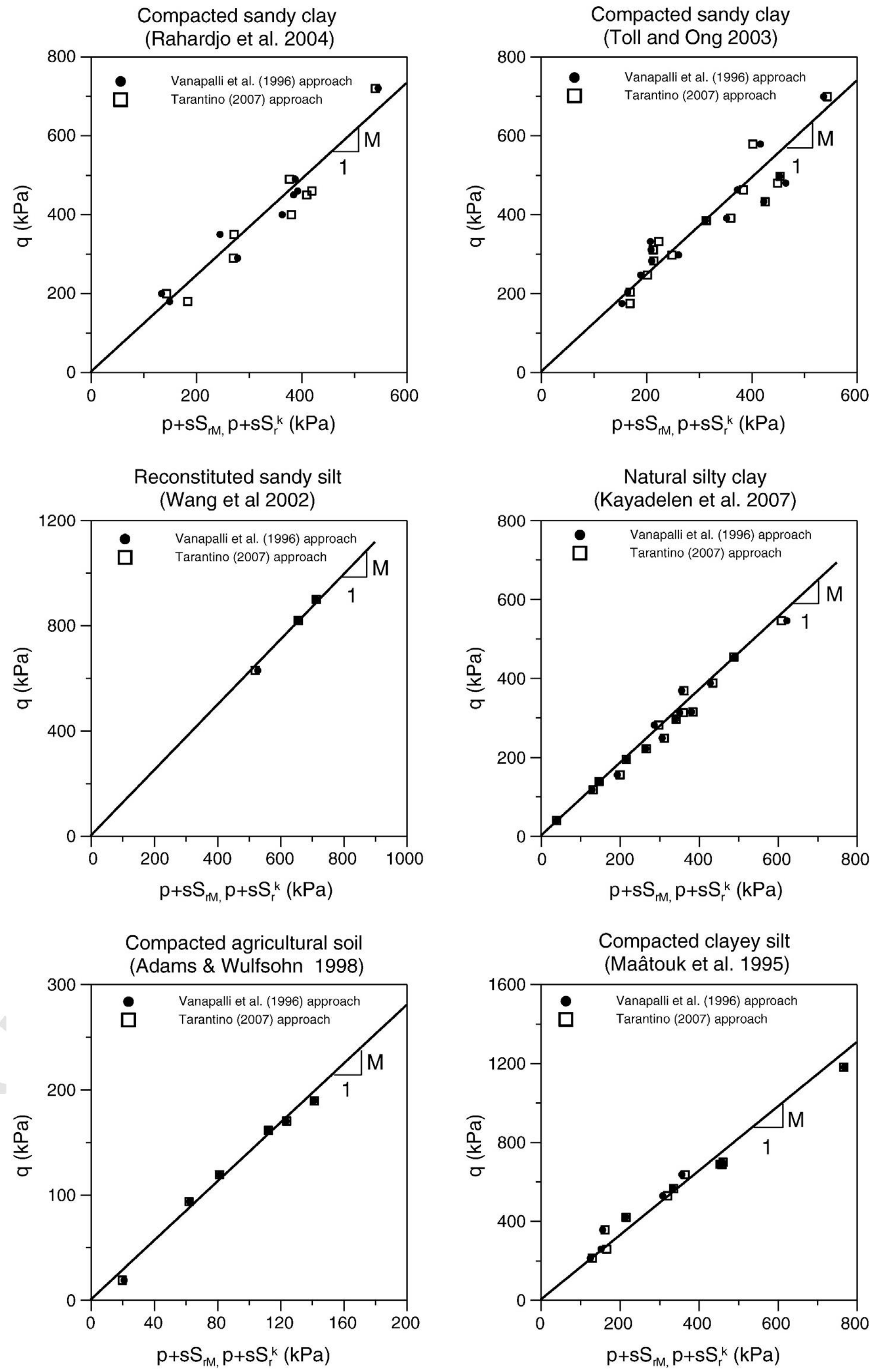

Fig. 3. Validation of Vanapalli et al. (1996) and Tarantino (2007) approaches against clayey 'aggregated' soils.

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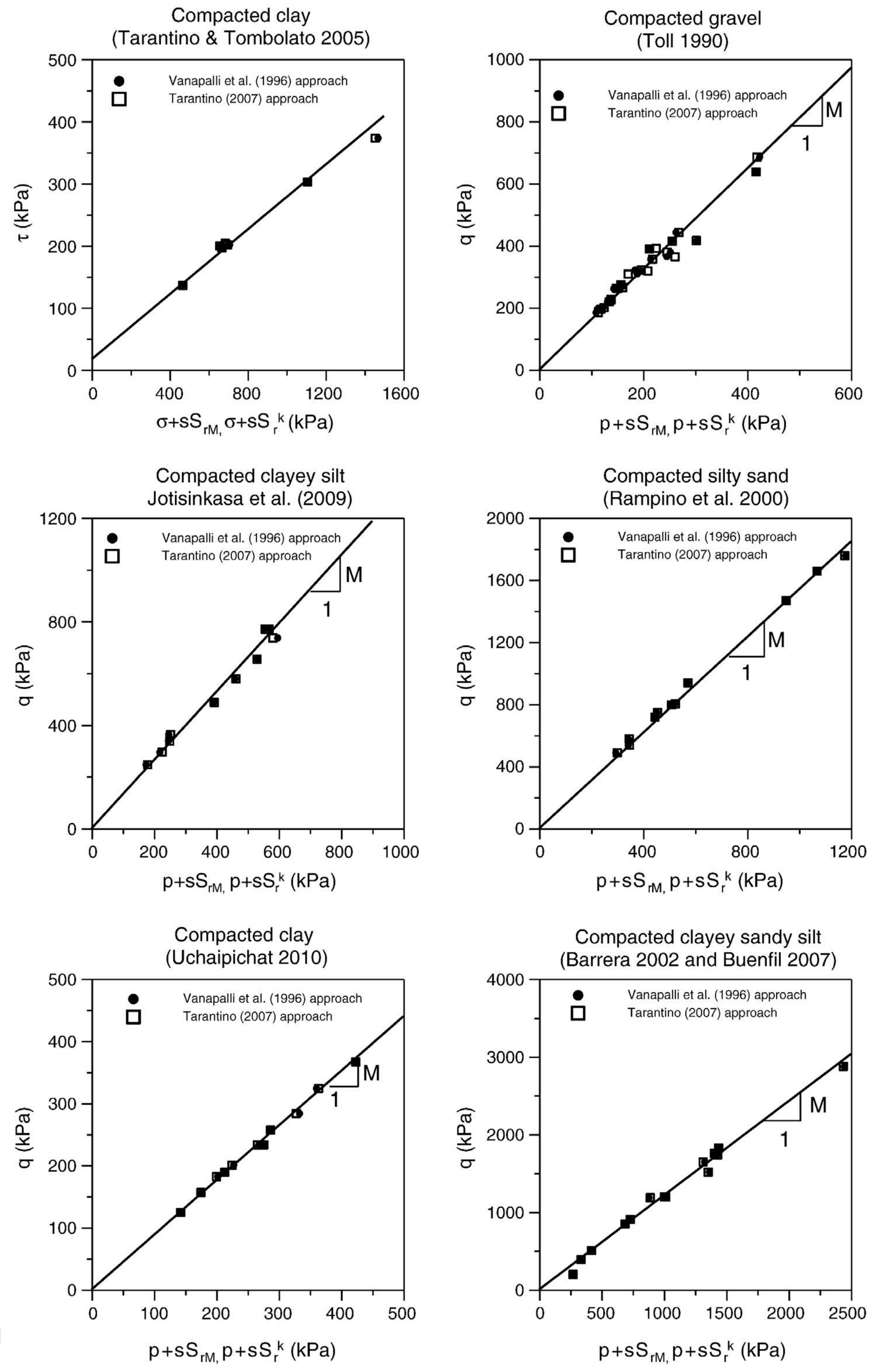

Fig. 3 (*continued*).

(1982) showed that α ranges from 1.1 for fine textures to 2.5 for coarse-grained materials. However, for the purpose of this investigation, a single value of $\alpha = 1.4$ was selected for all the soils studied here.

3.2. Validation of different approaches

The three methods were validated using soils with a variety of different textures including: (i) reconstituted, (ii) laboratory compacted, (iii) undisturbed specimens tested in the laboratory and (iv) in-situ soils. For each of these categories two soils were selected from the literature; the basic properties of these soils are presented in Table 2. Fig. 10 presents the estimation of the water retention behaviour of eight different soils using the three different methods described above in terms of degree of saturation and suction (kPa).

In most cases, the curve estimated using the Vereecken et al. (1989) empirical correlations results in a tailing off of the retention

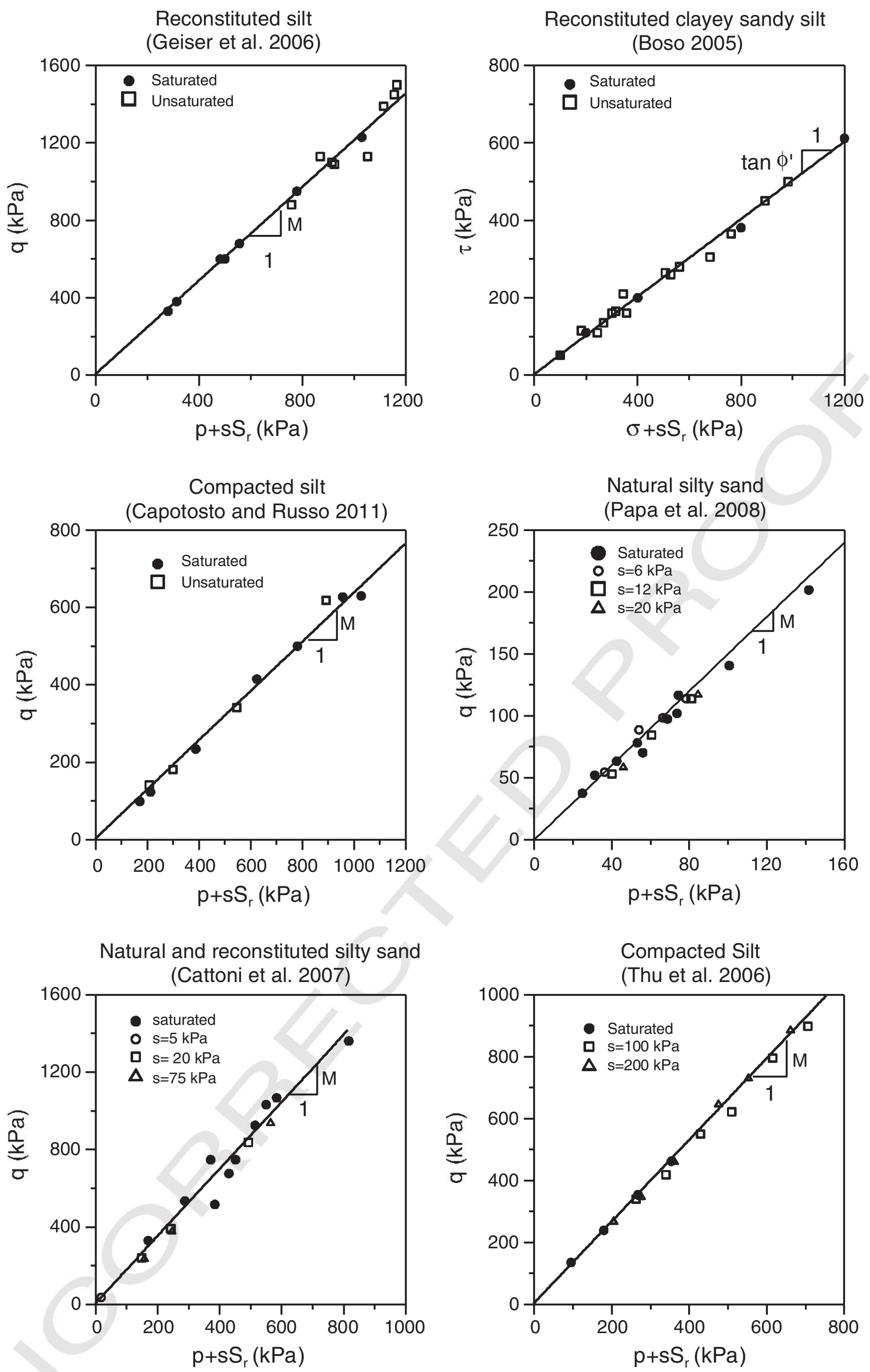

Fig. 4. Validation of Vanapalli et al. (1996) and Tarantino (2007) approaches against 'non-aggregated' soils ($e_{wm} = 1$ and $k = 1$).

curve at a fairly high degree of saturation at high suctions. This is due to an overestimation of the residual water content, θ_r, which is calculated based on the clay content and the carbon content:

$$\theta_r = 0.015 + 0.005(\%\mathrm{Clay}) + 0.014(\%\mathrm{Carbon}). \qquad (7)$$

This empirical equation was developed based on a database of soils having clay content between 2.3% and 19.6% whereas the soils tested in Fig. 10 have a clay fraction towards the upper end or outside of this range. The Vereecken estimation seems to fit well the experimental data for the undisturbed and the colluvium in-situ soil selected here. For the CDT in-situ soil (Fig. 10h), data is only available at very high degrees of saturation and it is therefore difficult to draw conclusions.

Fig. 13 shows that prediction of the WRC using the Vereecken method is poor for reconstituted soils (Fig. 10a, b) and ambiguous for compacted soils (Fig. 10c, d). This can be explained in terms of soil microstructure. Reconstituted soils have a microstructure markedly different from natural 'aggregated' soils used to validate the Vereecken's empirical model. On the other hand, compacted soils exhibit a variety of microstructures depending upon the moulding water content and compactive effort, which can sometimes be similar or sometimes different from that of natural 'aggregated' soils.

Where more detailed information on the grain-size distribution was not available the Arya and Paris method was used based only on the available sand, silt clay fractions (Fig. 10a, c, d, e). Despite this the Arya and Paris (1981) model was able to estimate reasonably well the retention curve behaviour for the compacted, undisturbed and in-situ

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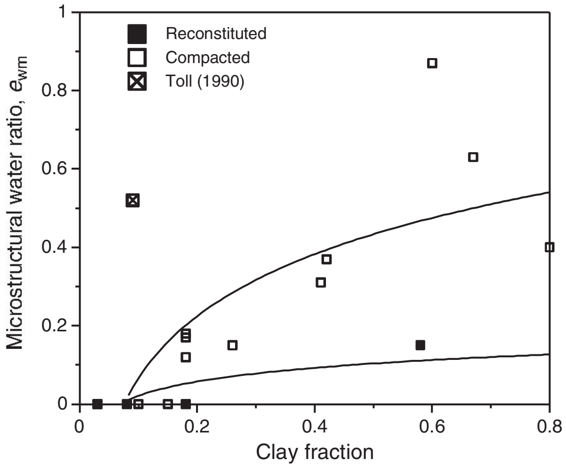

Fig. 5. Correlation between the microstructural water ratio e_{wm} as inferred from shear strength data and the clay fraction.

soils. However the air-entry value of the curve is less well defined using this method. Furthermore the Arya and Paris model significantly underestimates the capacity of reconstituted soil to retain moisture content at a given suction (Fig. 10a, b). This could be due to the selection of the α parameter, which was calibrated by Arya and Paris (1981) against loam-silty clay samples prepared as dry powder mixtures and then wetted by capillary rise. These samples would most likely exhibit an aggregated structure in contrast to reconstituted soils (prepared as a slurry and then consolidated under one-dimensional or isotropic conditions).

The retention curves estimated using the 'one-point method' (based only on moisture content measurement at a particular suction and void ratio) provide a very good estimation of the retention behaviour for all of the eight soils studied (Fig. 10), at least over the range for which experimental data is available. The full symbol (circle or square) indicates the point used to estimate the curve shown. Chin et al. (2010) recommend that for fine-grained soils this is at a suction of 500 kPa, where this measurement was not available the closest measurement was used. Despite the fact that Chin et al. (2010) used the retention curve of undisturbed soils to develop their empirical relations, the model is able to predict well the retention behaviour of fine-grained reconstituted, compacted, undisturbed and in-situ soils.

Due to the good performance of the Chin et al. (2010) method as illustrated in Fig. 10, it was investigated if the one-point method could capture additional features of water retention behaviour: (i) hydraulic hysteresis (Fig. 11), (ii) influence of stress level/void ratio (Fig. 12) and (iii) influence of soil fabric (Fig. 13). Fig. 11a, b and c illustrates that if one data point is obtained along a drying path and one point along a wetting path it is possible to assess the possible hydraulic hysteresis for the material under consideration without carrying out a full drying–wetting cycle, which for clayey soils is very time consuming and requires a number of different experimental techniques.

Fig. 12 illustrates that the one-point method can also be used to assess the influence of stress-induced void ratio change on water retention behaviour as illustrated for the Barcelona Silt (Fig. 11a) and Completely Decomposed Volcanic soil (Fig. 11b). The one-point method is also successful in accounting for differences in soil fabric. Fig. 13a presents water retention data for specimens prepared by

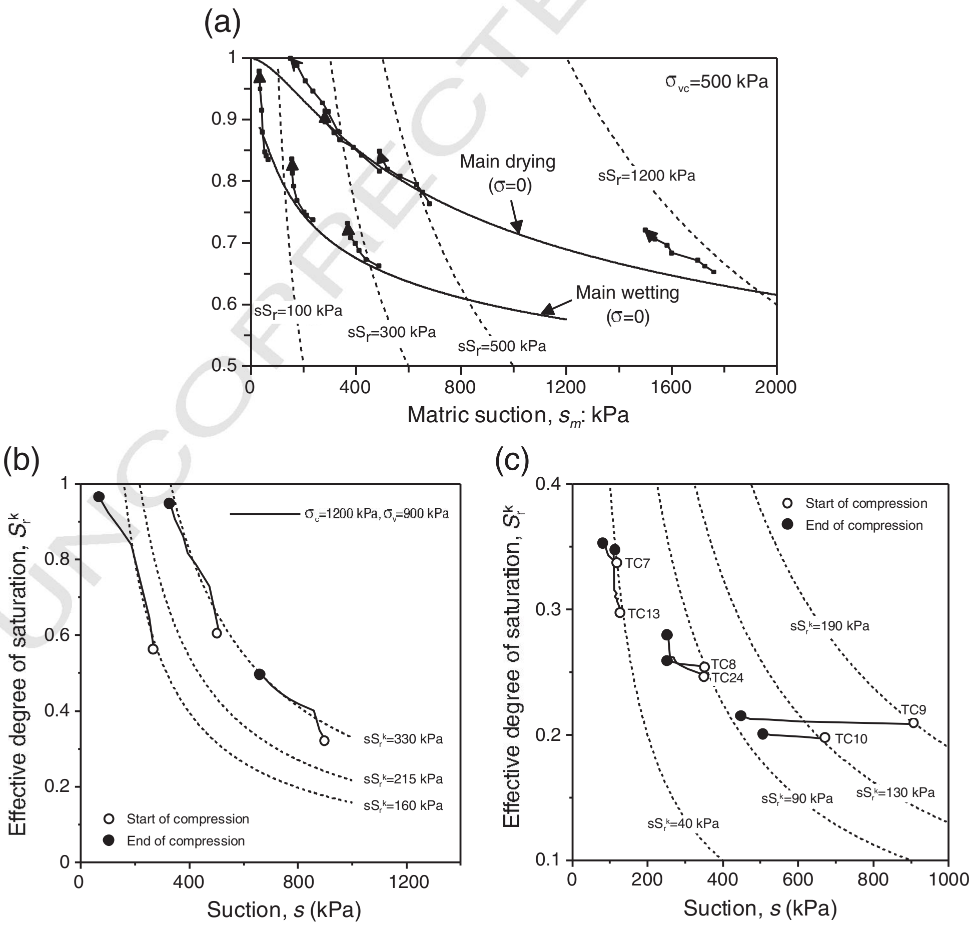

Fig. 6. Hydraulic paths of under water-undrained compression. (a) Reconstituted clayey sandy silt (Tarantino, 2009); (b) compacted clay (Tarantino and Tombolato, 2005); (c) compacted clayey silt (Jotisankasa, 2005).

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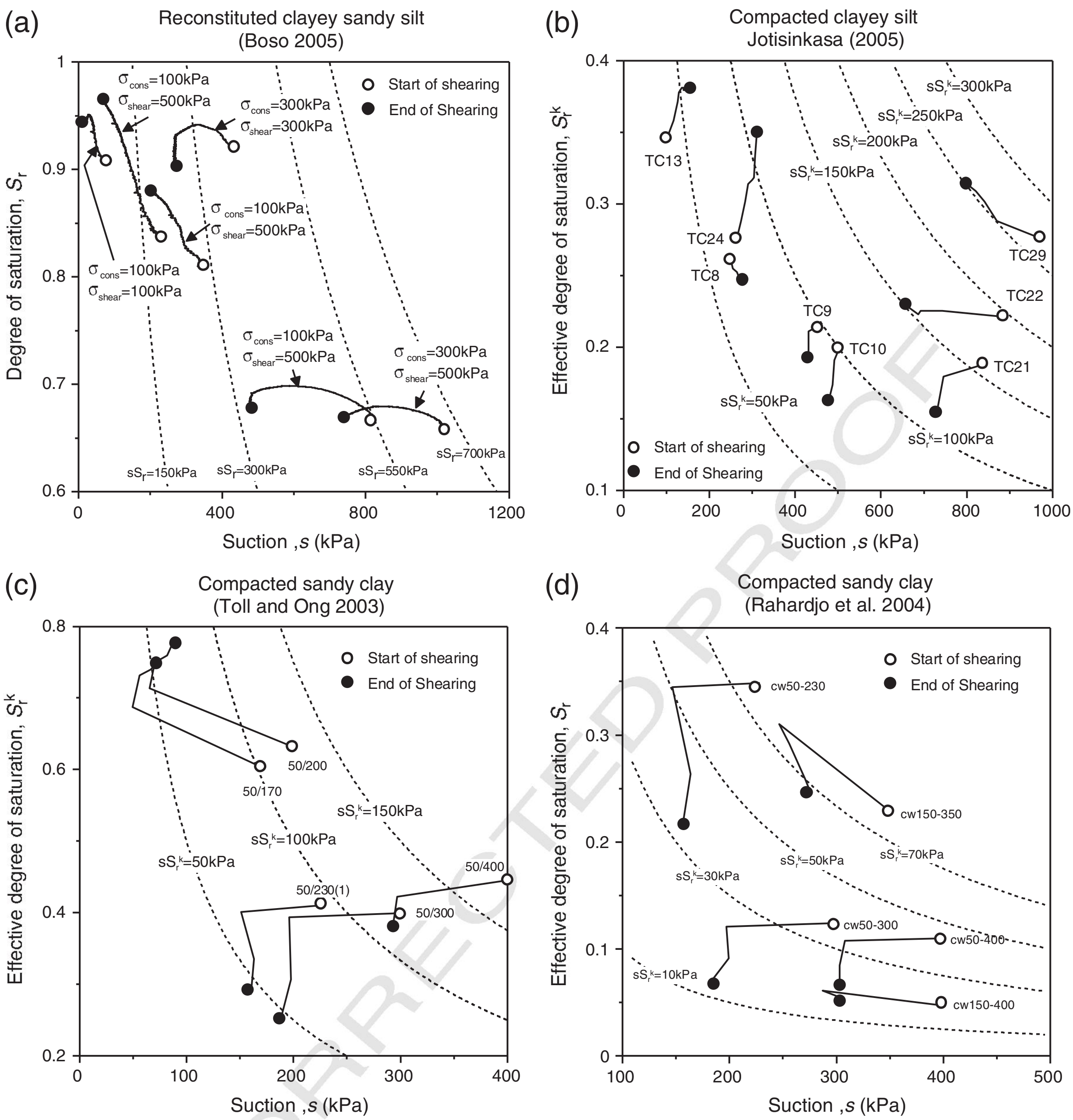

Fig. 7. Hydraulic paths of reconstituted and compacted and reconstituted soils upon water-undrained shearing. (a) Boso (2005); (b) Jotisankasa (2005); (c) Toll and Ong (2003); (d) Rahardjo et al. (2004).

compacting at similar initial void ratio but at different initial water contents on the dry and wet side respectively (Vanapalli et al., 1999). Fig. 13b presents a comparison between samples prepared from the same soil in either compacted or reconstituted state.

4. Accessible approach for preliminary assessment of the stability of geo-structures under unsaturated conditions

The previous sections have illustrated experimental procedures for 'low-cost' characterisation of shear strength and water retention behaviour. These can serve as a basis to perform preliminary stability and water flow analyses using simplified approaches that are illustrated in the following sections.

4.1. Stability analysis

The collapse behaviour of geotechnical structures can be analysed within the framework of limit analysis. The upper and lower bound theorems of plastic collapse set limits to the collapse load of a structure and can be proven for the case of perfectly plastic materials (Atkinson, 1981; Chen, 2007).

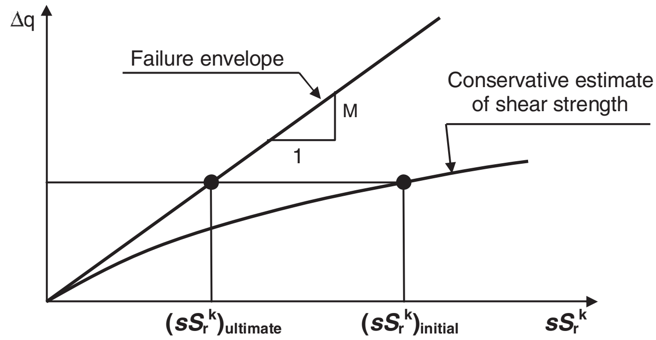

Fig. 8. Conservative estimate of shear strength based on water retention data at zero total stress and water-undrained shear test.

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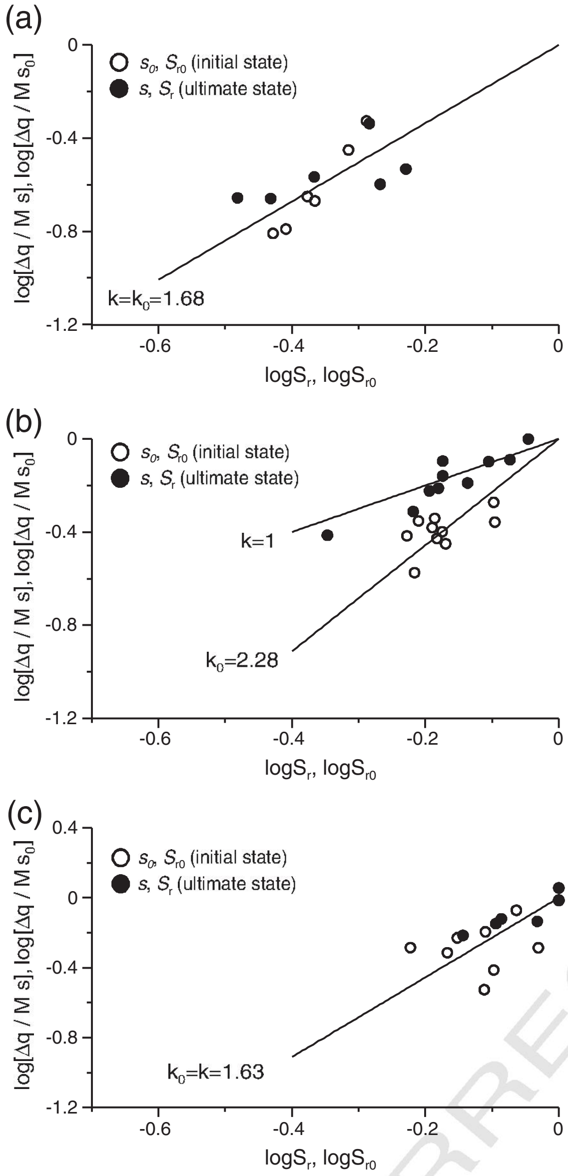

Fig. 9. Estimation of shear strength via the parameter k_0 based on water-undrained shear test and initial suction s_0 and degree of saturation S_{r0}. Parameter k is reported for comparison. (a) Compacted clayey silt (Jotisankasa et al., 2009); (b) reconstituted clayey sandy silt (Boso, 2005); (c) compacted clay (Tarantino and Tombolato, 2005).

In two-phase soils, the failure envelope under ultimate conditions can be defined by the following equation:

$$\tau = (\sigma - u)\tan\phi' \tag{8}$$

where τ is the shear stress, σ is the normal stress, u is the pore pressure and ϕ' is the effective angle of shearing resistance. Pore pressure equals the pore-water pressure u_w in saturated soils or the pore-air pressure u_a in ideally dry soils. Using the shear strength criterion given by Eq. (8), the ultimate conditions of soil structures such as retaining walls, foundations, vertical cuts, and slopes can be assessed for saturated and dry soils (Atkinson, 1981).

If Eqs. (3) and (5) are used in place of Eq. (8), collapse of geostructures in partially saturated soils can be analysed in a very similar manner by introducing a few simple modifications. To derive the upper bound solution, the work done by the internal stresses W_i can be written as (assuming an effective cohesion $c' = 0$):

$$W_i = \delta \sin\phi' \int_l s S_{re}\, dl \tag{9}$$

where δ is the magnitude of the block displacement, ϕ' is the effective (saturated) angle of shearing resistance, s is the suction, S_{re} is the effective degree of saturation ($S_{re} = S_{rM}$ or $S_{re} = S_r^k$), and l is the length of the failure surface. It is worth mentioning that the work done by the internal stresses W_i is written here in terms of total stresses whereas the external work associated with the gravitational load is calculated by considering the (total) soil unit weight.

To derive the lower bound solution, the failure criterion must not be exceeded at any point in the soil. This occurs if none of the Mohr's circles cross the failure envelope in the $\sigma + sS_{re}$, τ plane (rather than the σ', τ plane as in the case of saturated or dry soils). Examples of the application of limit analysis to the stability of geotechnical structures considering unsaturated soils are given by Stanier and Tarantino (2013), Amabile et al. (2012), and Balzano et al. (2012).

4.2. Analysis of rainfall effects

The beneficial effect of suction on the stability of geotechnical structures can partially or totally vanish when rainwater infiltrates at the ground surface. A simple method for preliminary assessment of the effects of infiltration rainwater on suction can be used, which is based upon solutions available in classical geotechnical textbooks focusing on saturated soils (Stanier and Tarantino, 2013).

As a first approximation, the water flow equation can be written by assuming a rigid soil skeleton and iso-thermal conditions, and neglecting water vapour flow. The water flow governing equation

Table 2
Basic soil properties of eight soils selected from the literature to assess methods for predicting the water retention function.

	Soil	State	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	w_L (%)	w_P (%)	G_s
a	Jossigny silt Fleureau et al. (1993)	Reconstituted	0	20	52	28	37	18	2.74
b	BCN silt Boso (2005)	Reconstituted	0	39.7	42.2	18.2	32	16	2.67
c	Jossigny silt Fleureau et al. (2002); *Cui and Delage (1996)	Compacted	0	4*	61*	35	37	16	2.74
d	Till Vanapalli et al. (1999)	Compacted	0	28	42	30	35.5	16.8	2.73
e	Jurong Rahardjo et al. (2001)	Undisturbed	0	22	39	39	48	24	2.64
f	CDV Ng and Pang (2000a, 2000b)	Undisturbed	4.9	20.1	36.6	37.1	55.4	33.4	2.62
g	Colluvium Ng et al. (2011)	In-situ	5	40	40	15	32	17	2.73
h	CDT Ng et al. (2011)	In-situ	0	25	60	15	34	20	2.68

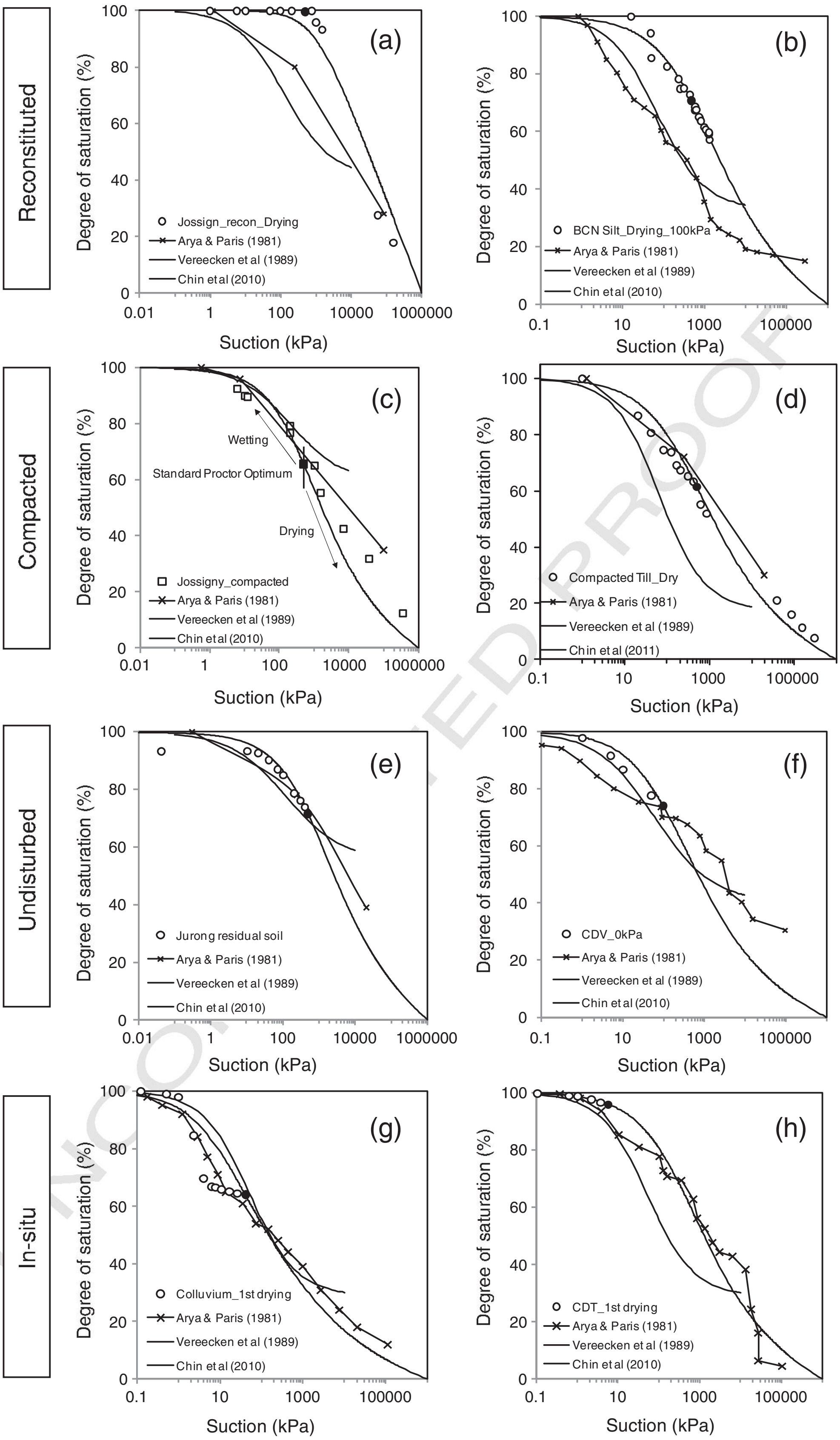

Fig. 10. Estimation of the water retention curves (drying) for eight soils using three methods.

can be linearised by assuming that i) the hydraulic conductivity is constant and ii) the water retention curve is linear. For conservatism the hydraulic conductivity is assumed to be equal to the saturated value as it returns the maximum possible infiltration and, hence, the highest reduction in suction and thus shear strength. Under these assumptions, the one-dimensional water flow equation becomes:

$$\underbrace{\left(\frac{}{\gamma\ \frac{\Delta\theta}{\Delta}}\right)}_{c_v}\frac{\partial^2 u_w}{\partial z^2}=\frac{\partial u_w}{\partial t} \tag{10}$$

where u_w is the pore-water pressure, t is the time, z is the vertical coordinate, k_{sat} is the saturated hydraulic conductivity, θ is the volumetric water content, γ_w is the unit weight of water, and $\Delta\theta/\Delta u_w$ is the slope of the linearised water retention curve. The term in parenthesis on the left-hand side of Eq. (10) is the equivalent of the consolidation coefficient c_v appearing in Terzaghi's consolidation equation for saturated soils.

Let us assume that the initial condition for pore-water pressure is hydrostatic and controlled by the groundwater table located at a depth H_w from the ground surface. This assumption is conservative as evaporation would significantly increase suction close to the ground surface. To simulate infiltrating rainwater, it is assumed for conservatism that ponded infiltration occurs, and therefore the hydraulic condition at the upper boundary is represented by zero pore-water pressure at the ground surface. Let us also assume a groundwater table at the bottom of the flow domain.

With these initial and boundary conditions, the problem reduces to the classical Terzaghi consolidation problem with triangular excess pore-water pressure and double-drainage. The solution of this problem is widely found in classical geotechnical textbooks:

$$u(z,t)=\sum_{n=1}^{\infty}\frac{2u_0}{n\pi}\left\{-\left(\frac{}{\pi}\right)\quad\pi\right\}\sin\frac{n\pi z}{2H}\exp\left(-\frac{\pi}{}\right) \tag{11}$$

where u is the excess pore water pressure, z is the vertical coordinate (positive upward), t is the time, u_0 is the initial excess pore-water pressure at the ground surface, H is the drainage path length, and T is the time factor ($T = c_v \cdot t / H^2$). Eq. (11) makes it possible to predict conservatively, the evolution of the suction profile with time and, hence, the calculation of the factor of safety over time.

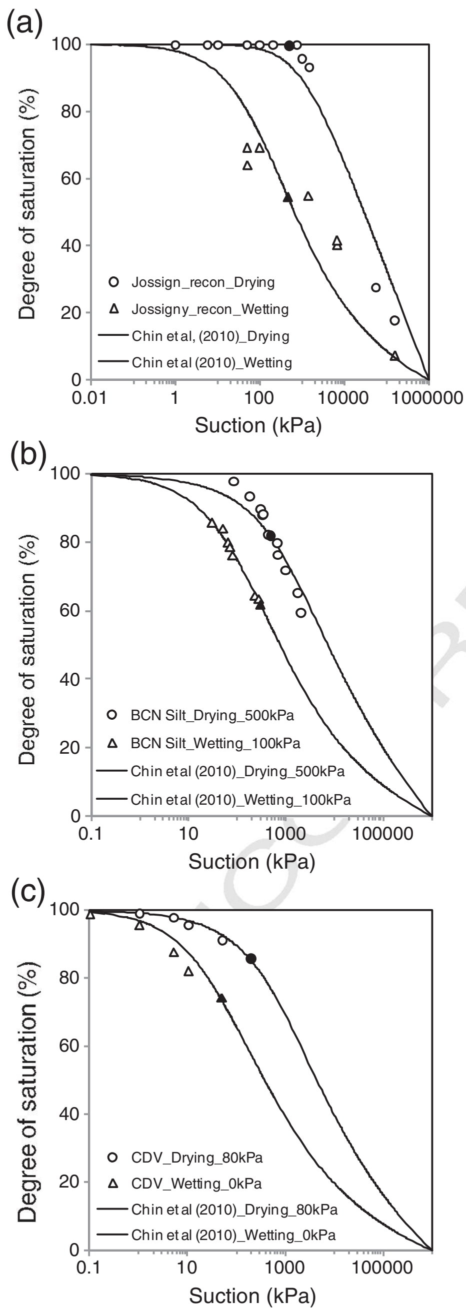

Fig. 11. Estimation of hydraulic hysteresis using the one-point method by Chin et al. (2010) for (a) reconstituted Jossigny silt, (b) reconstituted BCN silt and (c) undisturbed completely decomposed volcanic soil.

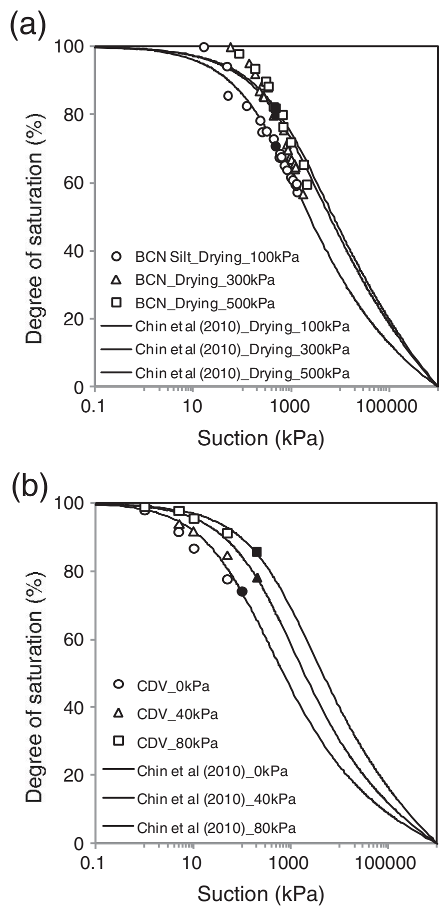

Fig. 12. The one-point method to capture the influence of stress level on water retention behaviour: (a) BCN Silt and (b) undisturbed completely decomposed volcanic soil.

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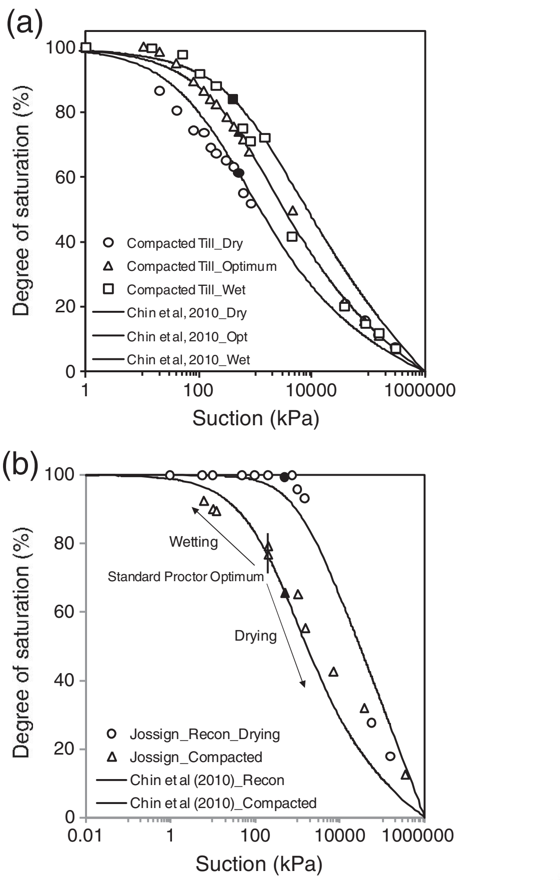

Fig. 13. The one-point method to capture differences in water retention behaviour due to differences in soil fabric (a) different fabrics created by compacting at moisture contents dry of optimum, at optimum and wet of optimum (Vanapalli et al., 1999) and (b) comparing the retention curves of reconstituted and compacted Jossigny silt (Fleureau et al., 1993, 2002).

5. An example of the preliminary stability assessment of a geotechnical structure

An application of our 'accessible' unsaturated soil mechanics approach is presented in this section with reference to the design of a cantilever retaining wall. The cantilever retaining wall designed in this exercise is shown in Fig. 14, with height $H = 5$ m and the water table located at a depth, $H_w = 10$ m. In routine practice, the retaining wall will be designed by assuming conservatively that the groundwater table is at the surface of the retained material (CEN, 2004) to account for saturation caused by infiltrating rainwater. This design will be here compared with the design derived by assuming the soil above the water table to be unsaturated and analysing conservatively the effect of rainfall on suction loss.

The wall will be verified in this exercise for sliding only, with no consideration given to overturning moment and, hence, the design of the toe, and no consideration will be given to the structural design of the wall. For this exercise, the pyroclastic soil 'Class A' in Bilotta et al. (2005) consisting of about 50% sand, 50% silt, and a negligible clay fraction was considered. This soil has been selected because of its 'granular' nature as it is often assumed that suction effects are negligible in granular soils.

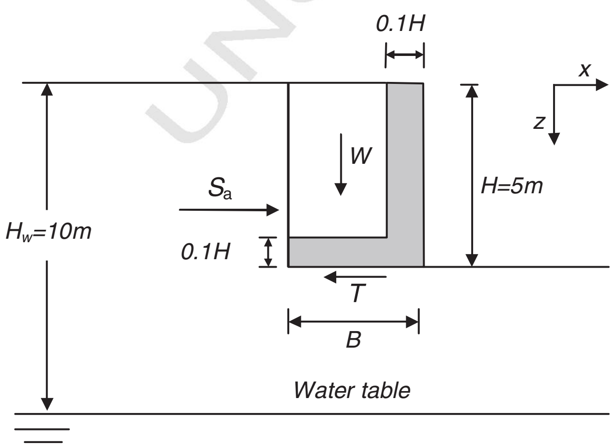

Fig. 14. Schematic layout of retaining cantilever wall.

5.1. Mechanical and hydraulic characterisation of pyroclastic soil

'Conventional' constant-water content tests are available for this pyroclastic soil (Bilotta et al., 2005). Samples having the same initial suction s_0 and degree of saturation S_{r0} were tested at different total normal stresses in the direct shear box. The contribution of partial saturation to shear strength $\Delta\tau = \sigma - \tan\phi'$ is plotted against the normal stress σ in Fig. 15. If the average value of $\Delta\tau$ is placed in Eq. (6), a value of $k_0 = 0.97 \cong 1$ is obtained. The shear strength criterion derived from constant-water content tests on this soil is therefore:

$$\tau = (\sigma + sS_r)\tan\phi'. \tag{12}$$

The water retention curve was estimated using the one-point method by Chin et al. (2010). A comparison between the curve fitting the experimental data as reported by Bilotta et al. (2005) and the curve estimated using the one-point method is given in Fig. 16, with the full symbol indicating the single point used to estimate the curve shown. It can be seen that the agreement is very satisfactory. The water retention curve (in the van Genuchten, 1980 form) adopted for this material is therefore:

$$S_r = \frac{1}{\left[\quad \alpha \quad\right]^m} \tag{13}$$

with $\alpha = 0.51$, $n = 1.18$, and $m = 0.15$.

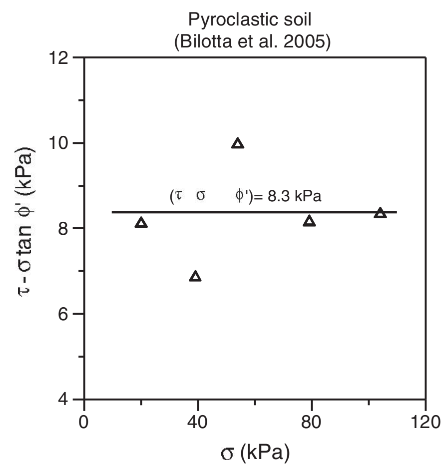

Fig. 15. Estimate of unsaturated shear strength of pyroclastic soil 'Class A' based on samples having the same initial suction and degree of saturation and sheared at different normal stresses under water-undrained conditions.
Data from Bilotta et al. (2005).

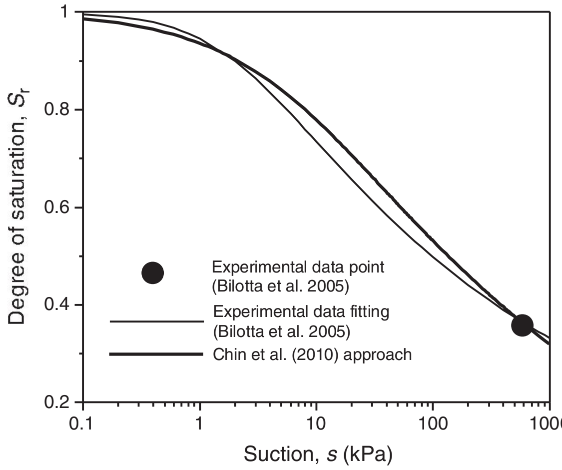

Fig. 16. Estimation of the water retention curve for pyroclastic soil 'Class A' (Bilotta et al., 2005) using Chin et al. (2010) approach (the full symbol indicates the point used to estimate the curve).

5.2. Estimation of active thrust on retaining wall

Stanier and Tarantino (2010) have shown that upper and lower bound solutions for the active thrust S_a are coincident. For this reason, only the lower bound theorem of plasticity will be considered in this example.

To derive the lower bound active thrust, firstly the state of stress in equilibrium with the applied loads needs to be considered and secondly it must be ensured that the stresses do not violate the failure criterion. If the vertical and horizontal directions are assumed to be principal directions of stress, the equilibrium stress state is given by:

$$\begin{cases} \sigma_z = \gamma z \\ \sigma_x = \sigma_x^0 \end{cases} \quad (14)$$

where σ_z and σ_x are the vertical and horizontal directions, respectively, and $\sigma_x{}^0$ is a constant. A lower bound value for the horizontal stress $\sigma_x{}^0$ is obtained by imposing the Mohr stress circle in the plane ($\sigma + sS_r$, τ) to be tangential to the failure envelope:

$$\sigma_x^0 + sS_r = k_a[\gamma \qquad] \quad (15)$$

where k_a is the active earth coefficient. The lower bound active thrust S_a can be obtained by integrating Eq. (15) over the height H:

$$S_a = \int_H \sigma_x^0 dz. \quad (16)$$

For the case of a hydrostatic suction profile:

$$s = \gamma_w (H_w - z) \quad (17)$$

the active thrust can be calculated by combining Eqs. (13), (15), (16), and (17). The active thrust should be counterbalanced by the shearing resistance T mobilised at the wall-ground interface:

$$T = [\qquad \cdot \quad - \qquad \gamma \quad] \tan\phi' + B\cdot(sS_r)_{z=H} \tan\phi'. \quad (18)$$

For the sake of simplicity, a perfectly rough interface with no slip between soil and concrete structure and sliding occurring inside the soil has been assumed in Eq. (18).

The active thrust and the wall dimensions for the case of groundwater table at the surface of the retained material (conservative design) and ground water table at $H_w = 10$ m with hydrostatic suction profile above water table are given in Table 3. It can be seen that a wall having a base $B = 4.1$ m is required to verify the wall against sliding when the groundwater table is assumed to be at the surface whereas no wall at all is required when the unsaturated shear strength is taken into account (the vertical cut is found to be stable).

Table 3
Retaining cantilever wall design.

	S_a (kN/m)	B (m)	H (m)
Groundwater table at the surface of retained material ($H_w = 0$)	141	4.1	5
Groundwater table at $H_w = 10$ m and unsaturated soil above water table (hydrostatic conditions)	~0	~0	~0
Unsaturated soil (t = 1 day, $\Delta\theta/\Delta u_w = 0.01$ kPa^{-1})	2.5	~0	~0
Unsaturated soil ($t = 2$ day, $\Delta\theta/\Delta u_w = 0.01$ kPa^{-1})	15.2	0.2	5
Unsaturated soil ($t = 1$ day, $\Delta\theta/\Delta u_w = 0.05$ kPa^{-1})	~0	~0	~0
Unsaturated soil ($t = 2$ day, $\Delta\theta/\Delta u_w = 0.05$ kPa^{-1})	~0	~0	~0

5.3. Preliminary evaluation of rainfall effects

To draw the suction profiles associated with ponded infiltration using Eq. (11), two possible linearisations as suggested in Fig. 17 were considered. By assuming $k_{sat} = 5 \cdot 10^{-6}$ m/s (Bilotta et al., 2005), $\Delta\theta/\Delta u_w$ equal to either 0.05 or 0.01 kPa^{-1} (see Fig. 17), and a rainfall duration of 1 or 2 days, the suction profiles are derived as shown in Fig. 18. The active thrusts and the wall dimensions for the case of rainwater infiltration are given in Table 3. It can be seen that rainwater infiltration, analysed using very conservative assumptions, does not erase suction and that a retaining wall does not seem to be required even when rainfall is taken into account. In this exercise, the assumption that the groundwater table is at the surface of the retained material seems to lead to significant over-design of the wall.

6. Conclusions

The paper has presented an approach for the preliminary characterisation of shear strength and water retention behaviour of unsaturated soils aimed at performing a preliminary analysis of the stability of geotechnical structures above the water table. The overall philosophy behind the 'accessible' approach to hydraulic and mechanical characterisation is that experimental testing is not removed but reduced to a minimum.

It has been demonstrated that a constant-water content test carried out in conventional equipment with no facilities to control/monitor suction can be used for a conservative estimate of shear strength. In principle, since the unsaturated shear strength criterion requires only one parameter in addition to the 'saturated' ones, a single constant-water content test would be sufficient to estimate the unsaturated shear

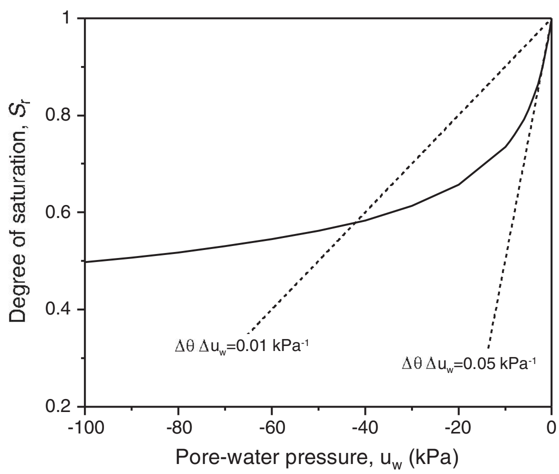

Fig. 17. Linearisation of the water retention curve.

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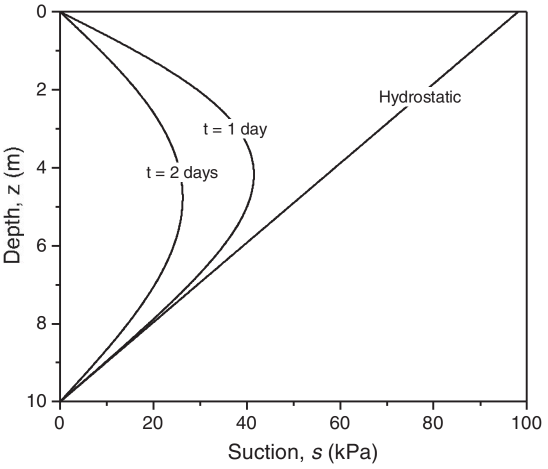

Fig. 18. Suction profiles from linearised equation for the case $\Delta\theta/\Delta u_w = 0.01\ \text{kPa}^{-1}$.

strength. As the degree of saturation and suction independently control the shear strength, the water retention curve(s) need to be estimated to predict shear strength in practical problems. It has been shown that the one-point method proposed by Chin et al. (2010), using a single measurement of moisture content (or degree of saturation) at a given suction provided a very good estimation of the water retention behaviour for all of the soils investigated with a variety of different soil textures. It appears that, it is the single measurement at a given suction which takes account in some way of the specific soil microstructure within the water retention curve estimation.

Finally, the stability analysis of a geotechnical structure considering unsaturated soils has been discussed within the framework of limit analysis. The critical element in 'unsaturated' design is the assessment of suction loss associated with infiltrating rainwater. A simple approach based on the linearisation of the governing flow equation has been proposed. Hydraulic parameters were chosen conservatively (the unsaturated hydraulic conductivity equal to the saturated one), and the initial and boundary conditions were also selected conservatively. The equation can be solved using classical solution from Terzaghi's theory of consolidation and in many cases, as shown in the example developed in this paper, can provide a useful preliminary indications to the geotechnical designer as to whether suction is playing an important role in the stability of a geostructures within a specific context.

Q6 7. Uncited references

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